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FLEXIBLE WELDED ANGLE CONNECTIONS*

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INTRODUCTION

THIS report presents the results of a series of tests made on welded beam-to-beam or beam-to-column connections as used in standard tier building construction. The connections are similar to some of those proposed as tentative standards in December 1939, by the American Institute of Steel Construction.[1] It was desired that the connections be flexible enough to allow with safety the full end rotation which might be expected in a freely supported simple beam.

Connections of the types shown in Fig. 1 (*a*), 1 (*b*) and 1 (*c*) were tested. The direct pull tests shown in Fig. 1 (*a*) were to determine the relative flexibility of different lengths of angle legs. On the basis of these direct pull tests the flexible top and seat angle connections shown in Fig. 1 (*b*) were designed for the simple beam end rotation of typical beam designs. The purpose of the top angle is simply to support the compression flange laterally. In the case of the beam web connections shown in Fig. 1 (*c*) the angles were intended to carry the end reaction as well as to provide flexibility and support against twisting at the ends of the beam.

Tests had previously been made at the Fritz Laboratory on seat and top angle connections similar to those in Fig. 1 (*b*). In these previous tests the top angles were much thicker than in the present series and the connections were designed to be "semi-rigid," or moment resisting.[2,3] There is currently much interest in the possibilities of the economical design made possible by the use of semi-rigid connections; nevertheless, most beams in buildings at present are designed with the assumption of simple supports and in such cases it is essential that the welded connections have the desired degree of flexibility to give full simple beam end rotation.

The present investigation was carried out at the Fritz Engineering Laboratory of Lehigh University, in cooperation with the Welding Research Committee of the AMERICAN WELDING SOCIETY. In October 1939, the Committee authorized this work and appropriated a sum of $200 to cover the cost of fabrication of specimens. The investigation was a regular research project of the Fritz Engineering Laboratory, of which Professor Hale Sutherland, Head of the Department of Civil Engineering, is Director. Acknowledgment is made to Mr. Howard J. Godfrey, Engineer of Tests, and to all others on the laboratory staff for their continued assistance in carrying out the program. Helpful suggestions regarding the program were made by Mr. Heath Lawson, Mr. La Motte Grover, Mr. F. H. Dill and others.

TEST PROGRAM AND PROCEDURE

The test program consisted of three groups of tests. Group I consisted of direct pull tests varying the angle leg size to obtain the relative flexibility of different leg lengths; Group II consisted of direct pull tests varying the weld on the outstanding leg and also subjecting the weld to repeated load; and Group III consisted of full-size connection tests designed on the basis of the results of Groups I and II, and subjecting the specimen to repeated loads. The direct pull tests in Groups I and II simulated the action of the top angle and the upper end of the web angle, permitting a selection of the most desirable angle size and welding procedure at a minimum of expense. Subjecting the angle to a direct pull tested it more rigorously than in the case of an actual top and seat, or web angle connection. Details of the size, type and method of fabrication of the test specimens are as follows:

Group I—The direct pull specimens (Fig. 1 (*a*)) were held during welding so that the welds on one pair of angles were all done with the legs in a vertical position and the bead laid horizontally, simulating the top angle leg welded to the column. The welds on the other pair of angles on the same specimen were all done with the legs in a horizontal position and the bead laid horizontally similar to a top angle leg welded to a beam. The specimen was jigged very carefully so that the two main pull plates were in a straight line. This group consisted of five specimens made up of equal leg angles $1/4$ inch thick and 4 inches long, the variable being the length of the leg. Table 1 presents the details of the specimens as well as test results.

Group II—These specimens, consisting of Tests No. 6 to 14 inclusive, were fabricated similarly to those of Group I. Five 4 by 4 by $1/4$-inch angle specimens and four $3\frac{1}{2}$ by $2\frac{1}{2}$ by $1/4$-inch angle specimens were tested. The type of the weld was varied in this group as illustrated in Table 2.

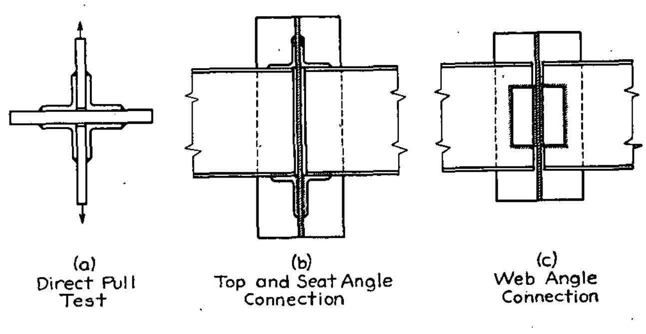

FIG. 1 TYPES OF FLEXIBLE CONNECTIONS IN TEST PROGRAM

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TABLE I ~ DIRECT PULL TESTS

TEST No.	ANGLE SIZE	WELD DETAIL	WELDING POSITION	YIELD POINT LOAD	YIELD POINT DEFL.	FIRST CRACK	MAX. LOAD	MAX. DEFL.
1	$4 \times 4 \times \frac{1}{4} \times 4$		Leg Vertical	4 100	0.140"	—	5 800	1.60"
			Leg Horizontal	3 700	0.112"	0.168"	4450	1.30"
2	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4} \times 4$		Leg Vertical	4100	0.086"	0.124"	5500	1.12"
			Leg Horizontal	4150	0.092"	—	5600	1.05"
3	$3 \times 3 \times \frac{1}{4} \times 4$		Leg Vertical	3 750	0.036"	—	6 550	1.02"
			Leg Horizontal	3 500	0.034"	0.302"	5 900	1.19"
4	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4} \times 4$		Leg Vertical	4575	0.022"	—	8500	0.83"
			Leg Horizontal	5 350	0.040"	0.160"	7800	0.80"
5	$2 \times 2 \times \frac{1}{4} \times 4$		Leg Vertical	5450	0.016"	—	9 150	0.59"
			Leg Horizontal	4 900	0.013"	0.098"	8 200	0.62"

Group III—This group of tests was made on six full size connections, of which three were top and seat angle connections, and three were web angle connections (Table 3). All these connections were fabricated at the Fritz Laboratory, using stub beam ends connected to the web of a 12 WF 65 stub column, as shown in Fig. 1 (*b*) and 1 (*c*). The connections were designed for the end reaction rotation corresponding to beam designs for three span lengths. A top and seat angle connection and a web angle connection was designed for each span length. A uniform D. L. + L. L. of 115 lb. per sq. ft. was assumed and calculations were based on the assumption that the beams would carry the entire load of a square floor panel having sides equal to span lengths of 20 ft., 22 ft. and 24 ft. The following beam sizes resulted: 16 WF 36, 18 WF 55, and 21 WF 59, respectively. All end shear was assumed as taken up by the seat angle in the top and seat angle connection. The seat angles were of the minimum size necessary to withstand the end reaction and were designed and welded in accordance with usual practice. In the case of the web angle connections the outstanding leg welds were designed to take the combined shear and bending stresses as in standard practice. It should be noted that the connections were tested with respect to rotation and did not carry end reactions corresponding to the actual design.

All welding was done at the Fritz Laboratory by a qualified welder, using a Grade 10 Electrode. The welds in every case were $^1/_4$-inch fillet, having the same size as the angle thickness. The angles were of stock size and were cut on a power saw to lengths of 4 in. ± $^1/_{16}$ in. for the direct pull specimens, and 6 in. ± $^1/_{16}$ in. for the full-size connection specimens. The flexible angle material conformed to A. S. T. M. Standard Specifications A9–36. The longitudinal edges of all connecting angles were welded in the as-rolled condition.

The gages for the direct pull tests in Groups I and II were mounted as shown in Fig. 2. Movement of the heel of the angle from the plate was measured with Ames Dials accurate to 0.001-inch. Beyond the gage range (1.0-inch) the deflection was measured with a steel scale graduated to 0.01 inch. A gage was placed on each end of each pair of angles and the movement of the heel was recorded and averaged.

The relative rotation between the beam end and the column in the Group III tests was measured by means of rotation bars attached to the members as in Fig. 3, which also shows the method of loading. The specimens were inverted and concentrated loads were applied to the beams to produce rotation of the beam ends. The rotations were measured by a 20-in. level bar of the same type used in a previous investigation.[3] The level bar was sensitive to a rotation of $^1/_{20,000}$th of a radian or to ±10 seconds, and consisted of a 10-second precision level bubble mounted on an aluminum bar. Two sharpened steel points supported the bar at one end, and the other end was supported by a micrometer screw which was used to bring the bar to level position for each reading. The elevation of the micrometer end of the bar was read by a $^1/_{1000}$ Ames Dial. The relative

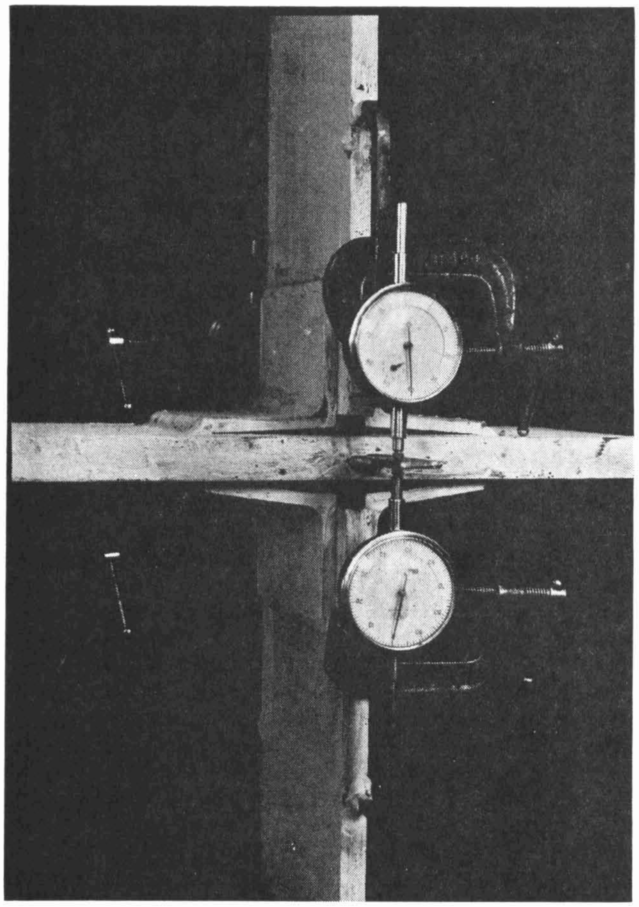


Fig. 2—Direct Pull Specimen, Test No. 4, After Yield

dial movement divided by the gage length gave the relative rotation in radians between any two load conditions. For each measurement the level bar rested in an identical location upon the polished surface of the rotation bars which were attached by arms to the beam and column. Ames Dials accurate to a movement of 0.001-inch were mounted on the upper and lower flange of the beam, bearing against the column, thereby enabling the location of the center of rotation.

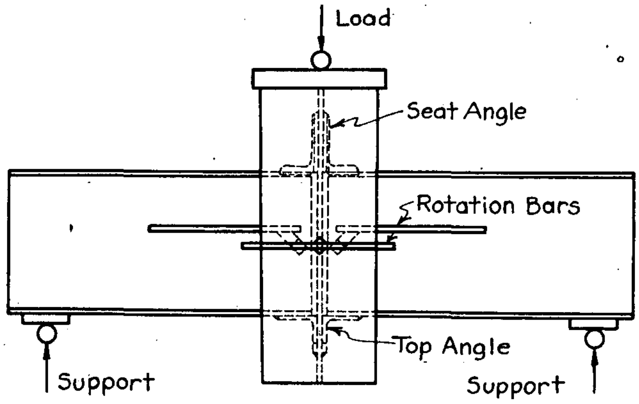
TEST RESULTS

Group I—Movement of the heel of the angle from the plate was measured at 200-lb. increments until the relation between load and deflection deviated markedly from a straight line. From then on this movement was recorded periodically in order to get the maximum deflection before actual fracture. At every 1000-lb. increment up to just beyond the approximate yield strength the load was dropped to the initial load, and record of the permanent set was obtained. A typical load deflection curve is shown in Fig. 4.

From the load deflection curve an arbitrary yield strength was established. This point was found in all cases by drawing a tangent to the straightest part of the curve in the low load range and a horizontal through the maximum load. The point where a vertical through the point of intersection of the first two lines crossed the curve was considered the yield strength. The relation between the yield strength and the length of outstanding leg is plotted in Fig. 5.

Flexibility was more important than strength and a preliminary study of Group I indicated that the 4 x 4 x $^1/_4$-inch angles would be suitable for a top angle and the $2^1/_2$ x $3^1/_2$ x $^1/_4$-inch angles suitable for web angle connections. These sizes were chosen on the basis of 0.10 and 0.08-inch heel deflections, respectively. The limiting heel deflections were below the general yield strength and below any noticeable local yielding or cracking in weld or angle. After making the actual beam connection tests in Group III the preliminary estimates were revised somewhat and limitations as to span length and beam depth are furnished in the summary.

Fig. 3—Beam Connection Test Arrangement



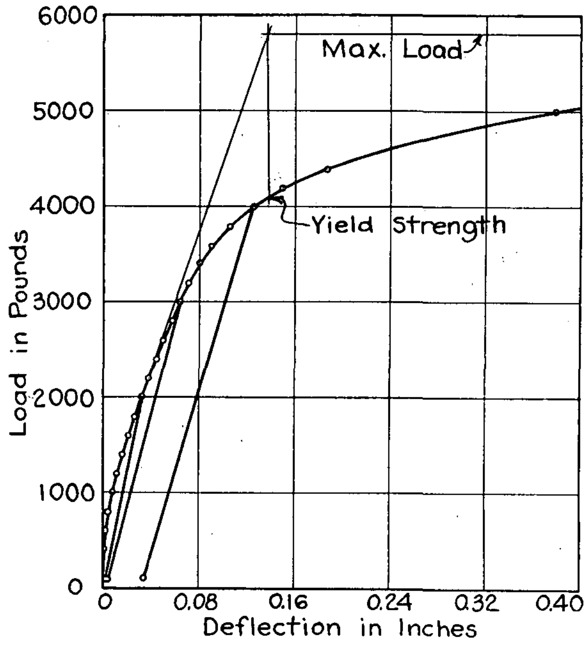


Fig. 4—Load Deflection Diagram Test No. 1, in Direct Pull

TABLE II DIRECT PULL TESTS UNDER REPEATED LOAD

Test No.	Angle Size	Weld Detail	Welding Position	Defl. at First Load Repetition	Load Range	No. Cycles	Creep	Defl. at Second Load Repetition	Load Range	No. Cycles	Creep	Defl. at Third Load Repetition	Load Range	No. Cycles	Creep	Max. Load	Max. Defl.	Description
6	$4 \times 4 \times \frac{1}{4} \times 4$		Leg Vertical	0.070"	200-3000	50	0.003"	0.112"	200-4000	40	0.003"	—	—	—	—	—	—	Not Loaded to failure
			Leg Horizontal	0.083"	200-3000	50	0.004"	0.180"	200-4000	40	Failed	—	—	—	—	4200	—	Yield due to repeated load
7	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4} \times 4$		Leg Vertical	0.045"	200-3000	100	0	0.075"	200-4000	100	0.005"	0.200"	200-5000	15	Failed	5000	0.593"	Yield due to repeated load
			Leg Horizontal	0.044"	200-3000	100	0.001"	0.075"	200-4000	100	0.006"	0.172"	200-5000	15	0.013"	—	—	Not loaded to failure
8	$4 \times 4 \times \frac{1}{4} \times 4$		Leg Vertical	0.092"	200-5000	100	0.007"	0.099"	2000-5000	55	0.003"	—	—	—	—	—	—	Not loaded to failure
			Leg Horizontal	0.109"	200-5000	100	0.033"	0.144"	2000-5000	55	0.009"	—	—	—	—	6100	0.420"	Yield due to repeated load
9	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4} \times 4$		Leg Vertical	0.056"	1000-4000	50	0	0.083"	1200-4800	50	0.006"	0.112"	1300-5200	50	0.006"	—	—	Not loaded to failure
			Leg Horizontal	0.058"	1000-4000	50	0	0.083"	1200-4800	50	0.008"	0.113"	1300-5200	50	0.012"	8800	0.526"	Failure due to static load
10	$4 \times 4 \times \frac{1}{4} \times 4$	1" 1"	Leg Vertical	0.075"	1200-4800	50	0.002"	0.092"	1300-5200	50	0.009"	—	—	—	—	8800	1.98"	Failure due to static load
			Leg Horizontal	0.086"	1200-4800	50	0.005"	0.138"	1300-5200	50	0.014"	—	—	—	—	—	—	Not loaded to failure
11	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4} \times 4$	1" 1"	Leg Vertical	0.052"	1300-5200	50	0.002"	0.070"	1500-6000	50	0.006"	—	—	—	—	—	—	Not loaded to failure
			Leg Horizontal	0.064"	1300-5200	50	0.004"	0.092"	1500-6000	50	0.008"	—	—	—	—	9500	1.60"	Failure due to static load
12	$4 \times 4 \times \frac{1}{4} \times 4$		Leg Vertical	0.079"	800-3200	50	0.003"	0.097"	900-3600	50	0.011"	0.126"	950-3800	20	0.013"	—	—	Not loaded to failure
			Leg Horizontal	0.085"	800-3200	50	0.001"	0.100"	900-3600	50	0.009"	0.123"	950-3800	20	0.603"	3800	1.40"	Yield due to repeated load
13	$4 \times 4 \times \frac{1}{4} \times 4$	$\frac{1}{2}$" $\frac{1}{2}$"	Leg Vertical	0.075"	1100-4400	50	0.005"	0.102"	1250-5000	50	0.014"	0.128"	1300-5200	50	0.030"	—	—	Not loaded to failure
			Leg Horizontal	0.079"	1100-4400	50	0.004"	0.103"	1250-5000	50	0.018"	0.139"	1300-5200	50	0.139"	5200	1.40"	Yield due to repeated load
14	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4} \times 4$	$\frac{1}{2}$" $\frac{1}{2}$"	Leg Vertical	0.056"	1400-5600	50	0.007"	0.072"	1500-6000	50	0.010"	0.099"	1600-6400	50	0.014"	—	—	Not loaded to failure
			Leg Horizontal	0.062"	1400-5600	50	0.006"	0.080"	1500-6000	50	0.010"	0.109"	1600-6400	50	0.015"	10000	0.877"	Failure due to static load

TABLE III BEAM CONNECTION TESTS

WEB ANGLE CONNECTIONS			TOP AND SEAT ANGLE CONNECTIONS				
20	19	18	17	16	15	TEST NO.	
$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}\times11$	$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}\times10$	$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}\times9$	$4\times4\times\frac{1}{4}\times6$	$4\times4\times\frac{1}{4}\times6$	$4\times4\times\frac{1}{4}\times6$	TOP ANGLE	
WEB A.S	WEB A.S	WEB A.S	10"×6" STIFFENED SEAT	$6\times4\times\frac{5}{8}\times8$	$7\times4\times\frac{7}{8}\times8$	SEAT ANGLE	
12WF65	12WF65	12WF65	12WF65	12WF65	12WF65	COLUMN	
21WF59	18WF55	16WF36	21WF59	18WF55	16WF36	BEAM	
	ALL	MOMENTS	IN INCH	POUNDS			
35,200	28,000	9,900	61,600	56,600	46,200	MOMENT AT FIRST REPETITION	
	ALL	ROTATIONS	IN RADIANS				
0.00465	0.00500	0.00495	0.00430	0.00525	0.00420	EAST BEAM	ROTATION
0.00465	0.00525	0.00325	0.00485	0.00450	0.00505	WEST BEAM	ROTATION
400-1600	350-1400	150-600	700-2800	700-2800	700-2800	LOAD RANGE	
50	50	50	50	50	50	NO. CYCLES	
0.00010	0.00010	0.00025	0.00040	0.00035	0.00035	EAST BEAM	CREEP
0.00005	0.00015	0.00015	0.00035	0.00035	0.00015	WEST BEAM	CREEP
41,800	34,000	11,550	68,200	62,000	49,500	MOMENT AT SECOND REPETITION	
0.00600	0.00670	0.00665	0.00590	0.00660	0.00600	EAST BEAM	ROTATION
0.00600	0.00695	0.00395	0.00675	0.00550	0.00740	WEST BEAM	ROTATION
475-1900	425-1700	175-700	775-3100	775-3100	750-3000	LOAD RANGE	
50	50	50	50	50	50	NO. CYCLES	
0.00020	0.00025	0.00020	0.00025	0.00060	0.00005	EAST BEAM	CREEP
0.00025	0.00020	0	0.00030	0.00045	0.00015	WEST BEAM	CREEP
48,400	36,000	13,200	72,700	66,000	59,400	MOMENT AT THIRD REPETITION	
0.00775	0.00745	0.00800	0.00695	0.00810	0.00705	EAST BEAM	ROTATION
0.00780	0.00785	0.00445	0.00780	0.00660	0.00845	WEST BEAM	ROTATION
550-2200	450-1800	200-800	825-3300	825-3300	900-3600	LOAD RANGE	
50	50	50	50	50	50	NO. CYCLES	
0.00025	0.00020	0	0.00020	0.00065	0.00040	EAST BEAM	CREEP
0.00020	0.00010	0.00005	0.00030	0.00040	0.00005	WEST BEAM	CREEP
70,400	56,000	46,200	136,400	92,000	89,100	MOMENT — EAST BEAM	FIRST VISIBLE CRACK IN RETURN WELD
0.01900	0.01760	0.02460	0.02675	0.01905	0.02340	ROTATION — EAST BEAM	FIRST VISIBLE CRACK IN RETURN WELD
70,400	56,000	46,200	145,200	92,000	82,500	MOMENT — WEST BEAM	FIRST VISIBLE CRACK IN RETURN WELD
0.01810	0.01820	0.02115	0.03825	0.01635	0.02180	ROTATION — WEST BEAM	FIRST VISIBLE CRACK IN RETURN WELD
0.00610	0.00652	0.00600	0.00610	0.00652	0.00600	DESIGN ROTATION	
0.03380	0.03035	0.03570	0.03165	0.04225	0.04360	EAST BEAM	MAX. ROT.
0.03470	0.03060	0.03295	0.03825	0.03555	0.04380	WEST BEAM	MAX. ROT.
88,000	68,000	56,100	145,200	128,000	103,900	MAXIMUM MOMENT	

Group II—As a result of the tests in Group I two angle sizes were selected for further direct pull tests in which variations in the weld details were tried out. The test procedure also included repetitions of load at 0.8, 1.0 and 1.2 times the desired total deflection corresponding to simple beam end rotation. Two sizes of angles were used in this group, 4 by 4 by $^1/_4$-inch angles to correspond to the top and seat angle connection, and $3^1/_2$ by $2^1/_2$ by $^1/_4$-inch angles, with the $3^1/_2$-inch leg outstanding, to correspond to the beam web connection.

Nine tests were made in all, and the results are tabulated in Table 2. In the first three tests, No. 6 to 8, the repeated load varied from the initial load at the zero increment to the loads necessary to give the previously mentioned total deflections. In these tests a perceptible creep during load repetitions was observed and the welds eventually fractured at 90, 215 and 115 repetitions, respectively. In every case, however, the deflection during the final load repetition was considerably more than necessary for the corresponding simple beam end rotation.

In tests Nos. 9 to 14, inclusive, different lengths of weld return, around the end of outstanding angle leg, were tried out. The repeated load range in these tests varied between the applied load and one-quarter of this amount, on the basis that some dead load is always acting. Two of these six tests failed as a result of load repetition. In the case of the other four tests, after having at least fifty repetitions at a deflection of twenty per cent or more in excess of the required, the connections withstood increased static load until total deflections of as high as 1.98 inch were reached at final failure.

A study of these test results indicated that a return of the weld around the ends of the angle toe equal in length to one-quarter of the length of the outstanding leg seemed to produce beneficial results. Tests 10 and 11, (Table 2), in which the 4-inch and $3^1/_2$-inch leg each have a return of 1 inch around the toe of the outstanding leg, show a very low creep at fifty repetitions of load producing deflections of 38 per cent and 15 per cent, respectively, in excess of the desired deflections. Upon the application of further steady load, initial yielding was found to be induced principally in the angle itself rather than in the root of the toe weld. The total deflection at final fracture was the greatest for this type of direct pull connection. Figures 6 and 7 show the condition at final fracture of Test 10 and Figs. 8 and 9 show corresponding pictures of Test 11.

Group III—Figure 1 shows the general weld details which were chosen for designing the beam connections in this group and the exact details of the six different tests are shown in Table 3. Rotations were measured at successive increments of load to the limit of the level bar's range. The distance between the heel of the angle and the column was measured thereafter with a steel scale. The load was repeated between one-quarter and full load fifty times each at 0.8 simple beam rotation, full simple beam rotation, and 1.2 simple beam rotation. A typical moment-rotation curve is shown in Fig. 11, which also shows the behavior which might

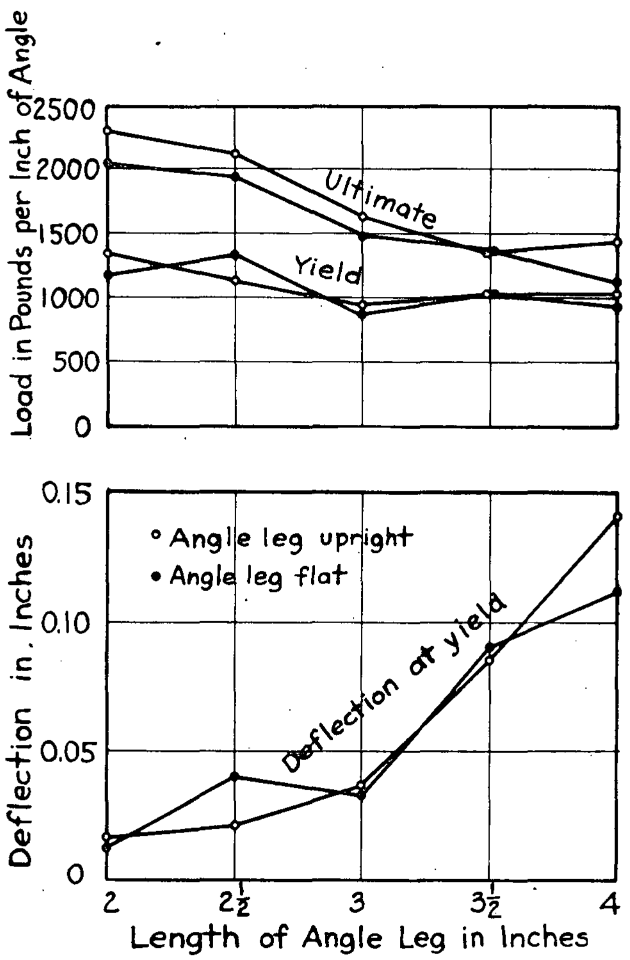


Fig. 5—Group I Test Results in Relation Angle Leg Length

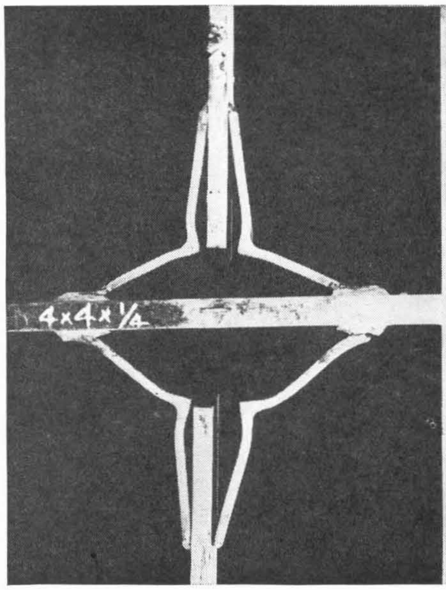


Fig. 6—Direct Pull Test No. 10, After Final Failure

Fig. 7—Direct Pull Test No. 10, After Final Failure

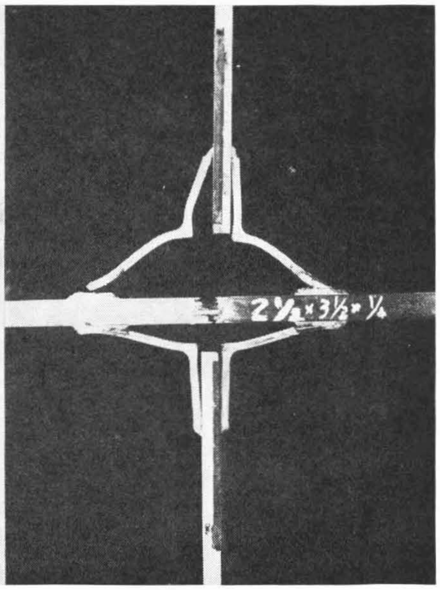


Fig. 8—Direct Pull Test No. 11, After Final Failure

have been predicted on the basis of the corresponding direct pull test No. 10.

The results of these tests confirmed the choice of weld details which had been made. At none of the three stages of load repetition was there appreciable creep. After completing the repeated load tests, each of the six connections took rotations far beyond full simple beam rotation to such an extent that it was not practicable in the testing machine to produce complete failure. The welds along the toe of the outstanding leg were not fractured in any of these tests, although partial tearing of the short weld returns took place. Figure 12 shows exposed views of Tests 15, 16 and 17, respectively, after stopping the test. Similar results were obtained in the case of the web angle connections, and these tests were stopped after the lower beam flanges came to bearing on the column web. Exposed views of these three tests are shown in Fig. 13.

SUMMARY AND CONCLUSIONS

The use of a short "weld return" on the cut edges of the connection angles as shown in Fig. 10 has been indicated to be beneficial in some respects. It does not follow, however, that connection angles without the weld return would be unsatisfactory. The tests in Group I may be used as a basis for the selection of angles without weld returns as it has been shown that the direct pull tests give a reasonably close prediction of the beam connection behavior. Arguments pro and con with respect to the use of a weld return may be summarized as follows:

In favor of the "weld return" or "boxing."

1. Initial failure of the material is forced into the angle rather than in the throat of the fillet weld.
2. High concentration of stress along the entire root of the top weld of a top angle is avoided.
3. Comparatively little additional weld metal is used.

In favor of omitting the "weld return."

1. An over-zealous welder could defeat the whole purpose of the design by running too far down the ends of the angles with the weld return.
2. Additional welding is introduced, increasing the cost.
3. The increased ultimate deflection of this type of connection is unimportant because the beam end could never rotate a fraction of the amount within the working range.

The limited number of repeated loads which were applied to the direct pull specimens and to the beam connections yield information regarding the capacity of the connections to take a limited number of overloads, but should not be construed to represent structural fatigue tests. The following conclusions are intended to apply to cases in which structural fatigue is not a problem.

A. *Direct Pull Tests.*—(1) The greatest strength and largest ultimate deflection prior to fracture was produced in the $3^1/_2$ and 4-inch outstanding legs when a return weld 1 in. long was carried around the toe of the angle on each side.

(2) Initial yielding and final failure of the connections with no weld return was in the throat of the weld.

(3) Weld returns as described in A-1 relieved the stress concentration in the root of the major portion of

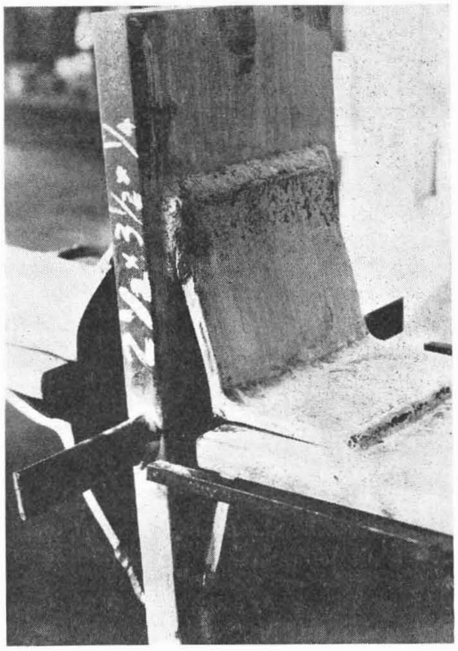


Fig. 9—Direct Pull Test No. 11, After Final Failure

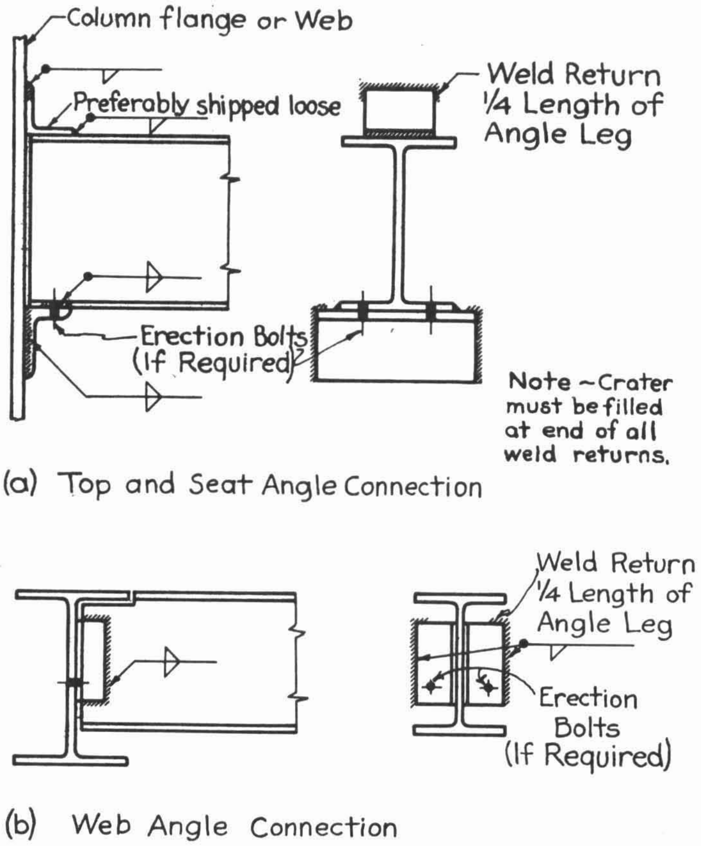


Fig. 10—Flexible Beam Connections

the fillet weld and caused most of the initial yielding to occur in the angle material. Final failure was by tearing of the weld returns, followed by fracture through the throat of the weld.

(4) The tests were not extensive enough to permit any definite conclusion regarding resistance of the connections to repeated overload.

B. *Beam Connection Tests*.—(1) The details shown in Fig. 10 are considered suitable for flexible welded angle connections for tier building construction. The top and seat angle type using a 4 by 4 by $^1/_4$ by 6-inch top angle, and the beam web connection using $3^1/_2$ by $2^1/_2$ by $^1/_4$ by ($^2/_3$ beam depth) will give satisfactory flexible connections within certain limitations of beam depth and span length. These limitations are based on limiting the angles with 4-in. outstanding legs to 0.10-in. heel deflection and the $3^1/_2$-inch legs to 0.08-inch heel deflection. The limitations may be summarized as follows:

(*a*) If the design is governed by a maximum deflection limitation of L/360, the maximum depth beam which should be used is 12 inches when the seat and top angle connection is used and 16 inches when the web angle connection is used.

(*b*) If the design is governed by maximum fiber stress of 20,000 lb. per sq. in. under uniform load, the maximum span length for top and seat angle connections is 19 feet and for beam and web angle connections 25 feet. For beams designed at a unit stress other than 20,000 these limits shall be modified by the ratio of $20{,}000/f_s$.

(2) In the top and seat angle type of connection the seat angle should be designed to take the full end reaction of the beam. The weld in the web angle con-

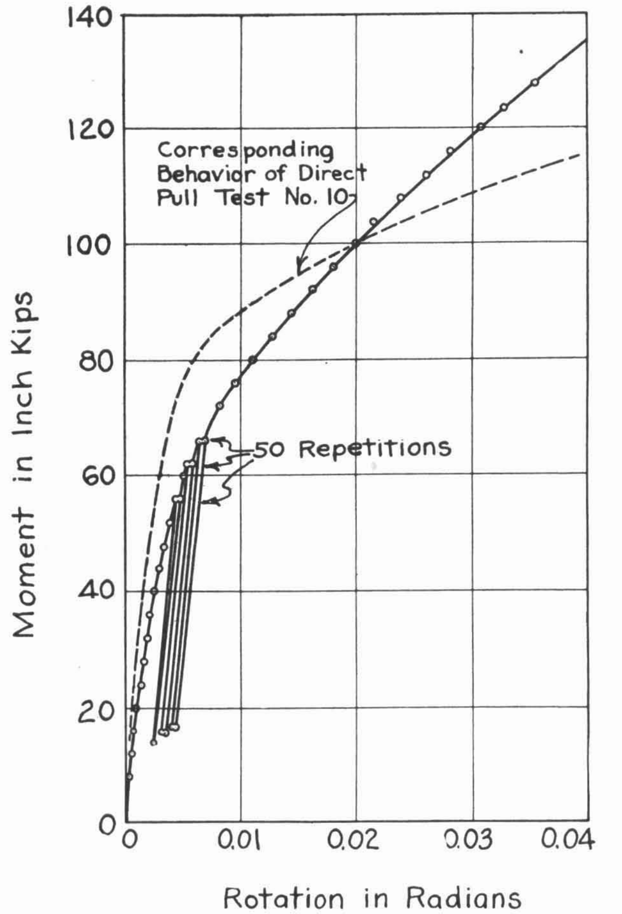


Fig. 11—Moment Rotation Diagram Test No. 16—Top and Seat Angle Beam Connection

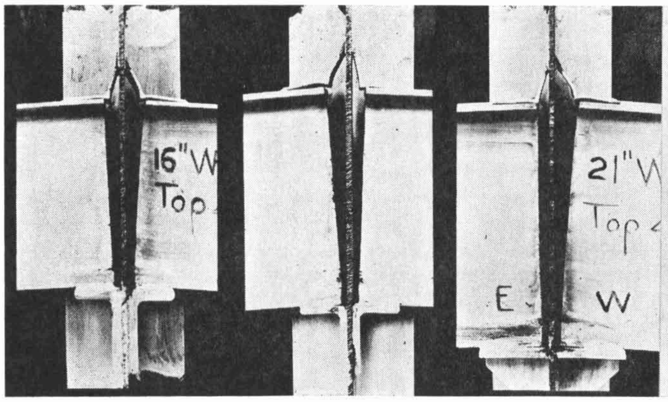


Fig. 12—Top and Seat Angle Connections, No. 15, 16 and 17, Respectively, After Testing

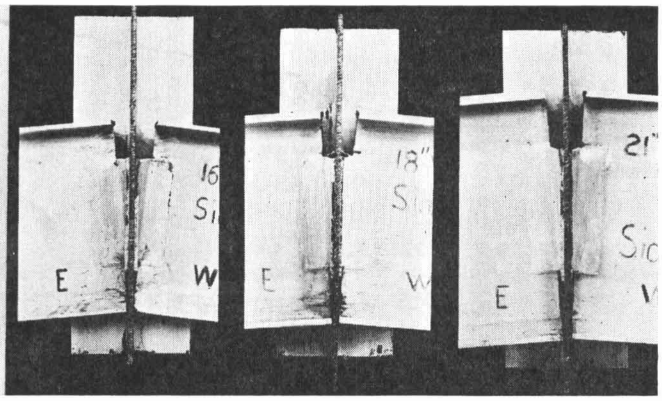


Fig. 13—Web Angle Connections, No. 18, 19 and 20, Respectively, After Testing

nection should be designed to take the stress due to combined shear and moment.

(3) The centers of connection rotation for the top and seat angle type were near the upper edge of the seat angle. The centers of rotation for the web angle connections were about three-quarters of the beam depth down from the upper edge of the beam.

(4) No substantial progressive increase in deflection was observed in the full size connections with a repeated load of one-quarter to maximum, repeating the load fifty times at eighty per cent of the design rotation, at design rotation, and at twenty per cent above design rotation.

(5) After completing the cycle of fifty-load repetitions at twenty per cent above design rotation, each connection continued to take increasing moment until rotations of more than three times the full simple beam rotation were reached. None of the connections had completely failed at this maximum rotation.

(6) The beam end moments which would be developed through the use of this type of connection would be less than ten per cent of the full fixed end moment in the range considered.

REFERENCES

1. Recommended Fundamental Principles, Tentative Minimum Requirements and Tentative Standard Welded Connections for Tier Building, American Institute of Steel Construction.
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